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STUDIES AND SPECIAL TESTING FOR HIGH EMBANKMENTS CONSTRUCTED WITH EARTH-ROCK MIXTURES

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AND FOUNDATION ENGINEERING IN MONTREAL, CANADA

SEPTEMBER 1965

STUDIES AND SPECIAL TESTING FOR HIGH EMBANKMENTS CONSTRUCTED WITH EARTH-ROCK MIXTURES

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SUMMARY

New highway routes through the mountains of California will necessitate the construction of several embankments approaching 400 feet in height.

The testing for use in design of high embankments involving soil-rock mixtures cannot be accommodated in conventional triaxial equipment. In an attempt to simulate embankment stress conditions on materials that were of the same types and sizes as contemplated for the prototype, large scale triaxial tests were conducted on 12-inch diameter by 28-inch high specimens of minus 3-inch material at confining pressures up to 8 atmospheres. Higher confining pressures to 27 atmospheres were also made but the specimen sizes were reduced to approximately 6-inches in diameter by 14-inches high, and the maximum particle size to 1½-inch. Tests were conducted to obtain both total and effective stresses. Degradation of the materials under compaction, consolidation and shearing were determined. The test data acquired has been used in design of one of the embankment structures with an overall height of approximately 385 feet.

STUDIES AND SPECIAL TESTING FOR HIGH EMBANKMENTS CONSTRUCTED WITH EARTH-ROCK MIXTURES

During the next few years it will be necessary to relocate many miles of Highway 101 in northern California. This route follows along Rattlesnake Creek and the Eel River. The terrain is very steep and much of the existing road is a two-lane road with rather sharp curves and low design speeds. The new route will be a four-lane highway with design speeds in the order of 60 miles per hour. In order to achieve these design standards, it will be necessary to construct several embankments that will be considerably higher than have been used in the construction of California highways. At least three creek crossings will have fills in excess of 350 feet high. As an alternative it would be possible to build structures at these locations, but the cost of the structures would be considerably greater than the cost of embankments. Due to the proximity of the new road to Rattlesnake Creek and to the Eel River, it will be necessary to use slopes no flatter than 2:1 and preferably as steep as 1½:1 in order to avoid encroachment of the embankment into the nearby creek or river. It was deemed necessary that studies should be made of the strength characteristics of the embankment material. Normally highway embankments in California have been constructed of roadway excavation with $1\frac{1}{2}$:1 or 2:1 slopes and heights as great as 100 to 150 feet. It was decided to undertake a rather comprehensive testing program using typical materials that would be available from roadway excavation along the projects to be constructed.

The materials in this area are derived from the large belt of sedimentary rocks of Upper Jurassic to Cretaceous Age. The sedimentary rocks consist chiefly of coarse sandstones, shales and minor conglomerates. Volcanic rocks are interbedded with sandstone and shale at a few places along the proposed alignment. Basalt, greenstone, chert and limestone are also present in minor quantities.

Three sites were selected for securing samples believed to be representative of the material that would be encountered in construction. A sample of material was secured from each of these sites. Each sample consisted of approximately three cubic yards of material. The material was excavated with a small shovel or loader and transported from the site to the laboratory in Sausalito, a distance of approximately 185 miles. In normal operations the material from these roadway cuts will be excavated by the use of loaders or rippers, with occasional blasting necessary in the zones of harder sandstone. One of the samples, probably the one of the poorest quality, was primarily shale in a slightly weathered condition. The second sample consisted of somewhat weathered fractured sandstone with some shale. The third sample was primarily a combination of sandstone and shale with somewhat more severe weathering than was involved in the other two samples. The testing reported herein was on this material. A fourth type of material will be available from roadway cuts to a limited degree. This material will consist of fairly fresh massive slightly fractured sandstone. It was not felt that it was necessary to test this sandstone since its strength will be considerably higher than any of the three samples tested. Several assumptions were made in formulating the testing program. First, it was assumed that the samples secured would be representative of the material that would be encountered in roadway excavation. Secondly, it was assumed that the materials could be aerated to the point where they could be readily placed. Third, it was assumed that some selection of material would be possible; hence, the material could be zoned using the poorer quality material in the areas where strength was not so critical and using the better quality material where the high strength was essential. Fourth, it was assumed that there would not be an appreciable increase in hydrostatic pressures in the material provided it was placed at optimum moisture content or somewhat dry of optimum.

The principal values that were desired from the testing program performed by the U. S. Army Engineer Division Laboratory at Sausalito, California, were shear strengths obtained under varying molding moisture-density conditions and drainage. Further, these values were needed on materials containing appreciable quantities of large gravel sizes and under test conditions of high confining pressures. Of the four previously described sources, the results presented herein were limited to the extensive testing of only one. In addition to the triaxial compression tests for strength determinations, permeability tests were performed on large specimens under varying confining pressures to determine the "k" value with change in void ratio as well as to observe the degree of degradation with change in confining pressure. This additional inclusion of permeability tests was immediately apparent when the magnitude of degradation was observed from the initial shear specimens. A study of degradation was made for the purpose of isolating the effect of compaction, consolidation, and shear, and may be noted on Figures 1 and 2. The percent compaction as noted on the various figures is a percent of a maximum as established from impact effort and is the Corps of Engineers Military Standard 621 CE55.

Three types of triaxial compression tests were conducted. The majority of tests were the "Q", or unconsolidated undrained type, in which the specimens were sheared at predetermined molding moistures and densities upon application of the confining or lateral pressure and without admission of any additional water. A limited number of tests designated as "R" modified were made to detect the possible increase in strength from consolidation prior to shear. No additional water was added to these specimens to gain saturation. The conventional "R", or consolidated undrained tests, were conducted on remolded specimens which had been saturated and consolidated prior to shearing. All three types of tests were conducted with closed systems in which no water was allowed to drain from the specimen during shearing action. Figures 1 and 2 illustrate the field grading of the minus 3-inch and minus 1½-inch portions of the material tested. All specimens tested at lateral pressures between 1 and 8.5 atmospheres or less contained minus 3-inch particle sizes. Between lateral pressures of 8.5 (125 psi) and 27 atmospheres (400 psi), the maximum particle size was minus 1½-inch. Subsequent to the completion of this testing program, a newly designed large scale triaxial device was placed in operation capable of testing 12-inch diameter specimens to lateral pressures of 34 atmospheres (500 psi).

The specimen size for minus 3-inch material was 12-inch diameter by 27.6-inches high and the minus 1½-inch material was accommodated in a 6-inch diameter by 13.8-inch high specimen. In the 12-inch diameter apparatus, [2, 4] the pore pressure was measured at seven different locations along the central axis of the specimens. Five of the sensing points were within the specimen and one at either end. The minute coaxial cables to the transducers in the specimens were brought through the rubber membrane at the elevation of the transducer and a minimum of miniature cable was actually within the specimen. The cables were then lead through the base of the apparatus to a measuring console. The volume of each transducer was about 0.0015 cubic feet and cylindrical in shape. The apparatus used for the minus 1½-inch material at the higher lateral pressures was developed by a private firm.[5] In this apparatus, the pore pressure was sensed only at the ends of the specimen. A drawing of the apparatus that accommodated the 12-inch diameter specimens is illustrated on Figure 3.

In defining the extent of the testing program, the method of tests was somewhat described. All tests were performed under a controlled rate of strain. For the "Q" test, the rate was established by observing all of the pore pressure cells simultaneously to detect the possibility of a pore pressure gradient developing. The tests were conducted as fast as possible and still permitting the pore pressure to remain equal throughout the column of soil. This permitted speeds varying between 3/4 and 1% per minute. For all "R" tests, it was necessary to reduce the speed to about 0.2% per minute. The test was stopped when both the maximum deviator stress and the maximum effective principal stress ratio had been reached. Volume changes during the shearing action were noted on pressure burettes connected to the water surrounding the specimen in the pressure chamber. One of the principal difficulties experienced in large scale and high pressure triaxial testing is to fabricate membranes that will withstand these pressures and still bridge angular surface interstices without developing membrane restraints that are noticeably significant. On the specimens tested at the two highest lateral pressures in the 12-inch diameter apparatus, it was necessary to use double membranes. On several of the 6-inch diameter specimens, it was necessary to employ a series of overlapping polyethylene strips about 2½-inches wide placed vertically between the two membranes. The thickness of the polyethylene strips was varied with the angularity of the material and the depth of the surface interstices.

Figure 1 shows the degradation results of a large scale triaxial test performed with a lateral pressure of 125 psi. The large degree of degradation resulting only from compaction is somewhat the exception and not the rule. On Figure 2, it will be noted that the largest degree of degradation was attained for the specimen compacted 2% dry of optimum. This is in direct relationship to the magnitude of energy expended during the molding operations as the drier the material, the more effort expended to meet a specific density. To demonstrate that degradation cannot be predicted with any degree of accuracy in a material of this type, it may be noted that the "R" modified test which was tested with only placement moisture shows a greater degradation than the standard "R" test which was totally saturated.

The permeability rate for a specimen molded to 93% compaction and at optimum moisture was 1×10^{-2} cm/sec. or about 30 feet per day. After consolidation with a confining pressure of 400 psi, the permeability rate was reduced to 1.3×10^{-3} or about 3.5 feet per day. It is, of course, impossible to separate the degradation due to compaction from that which resulted from consolidation. It would appear, however, from observing the results on figure 2 that about 10% of the gravel sizes (plus No. 4) were reduced to minus No. 4 particles when the permeability test at the 400 psi confining pressure was made.

The data on figure 4 are presented to show the change in pore pressure with variation in placement moisture at comparable confining pressures. It further serves to demonstrate the magnitude of pore pressure when compared to a specific confining pressure. The percent compaction was the singular factor common to all specimens. The high pore pressure value for the "R" test with a confining pressure of 400 psi is not unexpected as it represents the only test that was saturated before shearing. In comparing the "R" modified test at optimum moisture and chamber pressure of 400 psi with the comparable "Q" test, it may be observed that the pore pressure is somewhat higher in the "Q" test. This is a result of an appreciably larger void ratio at failure which resulted in a dissipation of the pore pressure since both specimens had the same placement moisture content.

The mohr failure envelopes for total stress and effective stress values are presented on figures 5 and 6 respectively. The letter designation for the various envelopes is common to both figures and the test conditions for each envelope are listed in table 1. The envelopes were developed from a minimum of three tests at confining pressures ranging between 1 (15 psi) and 27 atmospheres (400 psi) and, in some cases, as many as five tests covered the same confining pressure ranges. As anticipated, the highest strength envelope from the total stress plot was developed on materials prepared 2% dry of optimum and to 93% compaction. The lowest strength was at the lowest density investigated (85% compaction) and 2% wet of optimum. This produced a range of ϕ of 14° to 34°. The angles of the effective mohr failure envelopes as noted on figure 6 bracket a much smaller range of 30° to 38°.

Planning of the road in this area has proceeded to the point where the design of one of the relatively high embankments is nearing completion. This embankment will cross Squaw Creek with a maximum height slightly in excess of 350 feet. Geologic studies and boring programs were undertaken in order to evaluate the material that would be available for construction of the embankment across Squaw Creek and insure that the testing program had included the available material.

These studies consisted of geologic mapping, seismic surveys, and vertical and horizontal core and auger borings made in cuts in the vicinity of this embankment. From these data it was possible to determine roughly the percentages of the various types of material that would be available for embankment construction. The materials available could be classed in three general categories. The first category consisted of overburden and severely weathered material with a moderate to high percentage of clay as well as silt, sand, and rock sizes. The second category consisted of somewhat weathered sandstone and shale of fair to good quality. The third category of material was primarily relatively fresh, but somewhat fractured and jointed sandstone.

Using the data available from this exploration and the testing program described above a study was made of the possible zoning of materials in the embankment and the stability of several typical sections was calculated. The following strength values, based on data from the testing program, were used in the various zones in the proposed section.

Zone	Cohesion, lb./sq.ft.	Angle of Friction, Degrees	Percent Compaction
A	500	15	90
B	500	25	93
C	0*	35*	95*

*Estimated values

Stability analyses were made with two computer programs that had been developed or modified for use on the IBM-704 electronic computer. It was evident from the borings and geologic investigation that there would be a shortage of the best quality material, that is, sandstone from the cuts; hence, the cross-section of the sandstone portion of the proposed embankment was kept to a minimum that would still result in a stable section. The proposed cross-section for the embankment at Squaw Creek is shown on Figure 7.

Considerable selection of material from roadway excavation will be necessary in order to construct an embankment with the proposed cross-section. This will necessitate long hauls and stockpiling of material in order to insure its placement at the proper location in the embankment.

This paper demonstrates the need for relating sampling, exploration and testing of a combination of soil and rock to a specific problem in engineering design and construction. It is believed that the information obtained from the types of special apparatus used and from the magnitude of the confining pressures employed will be of immediate value to other professional people in the field of soil mechanics.

Acknowledgment is given to Mr. A. W. Root, Supervising Materials and Research Engineer, Division of Highways (retired May 1, 1962) under whose direction the program was planned and prosecuted and to Mr. Harry Cedergren, Senior Materials and Research Engineer, who was directly responsible for much of the planning and evaluation of the testing program. At the inception of the program, Mr. F. N. Hveem (retired) was Materials and

Research Engineer, a position presently held by Mr. John L. Beaton. The testing program was completed for the Division of Highways, State of California, by the Soils Section, U. S. Army Engineer Division Laboratory, South Pacific, which is under the general direction of Mr. C. B. Palmer, Director. Assistance of Mr. Melvin W. Cohen in preparing portions of the test data and his excellent work in performing the triaxial testing is appreciatively acknowledged.

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TABLE 1 - TEST DATA FOR CURVES ON FIGURE NO. 4

Curve No.	Type of Test	Lateral Pressure psi	Percent Compaction	Specimen Molding Moisture
1	Q	15	93	Optimum +2%
2	Q	60	93	Optimum +2%
3	Q	125	93	Optimum +2%
4	Q	200	93	Optimum +2%
5	Q	400	93	Optimum +2%
6	Q	400	93	Optimum -2%
7	Q	400	93	Optimum
8	R	400	93	Optimum
9	R (Modified)	400	93	Optimum

TABLE 2 - TEST DATA FOR FAILURE ENVELOPES ON FIGURES 5 AND 6

Envelope No.	Type of Test	Lateral Pressure psi	Percent Compaction	Specimen Molding Moisture
A	Q	400	85	Optimum +2%
B	Q	400	88	Optimum
C	Q	400	85	Optimum -3%
D	Q	400	93	Optimum +2%
E	Q	400	93	Optimum
F	Q	400	93	Optimum -2%
G	R (Modified)	400	93	Optimum +2%
H	R (Modified)	400	93	Optimum
I	R	400	93	Optimum

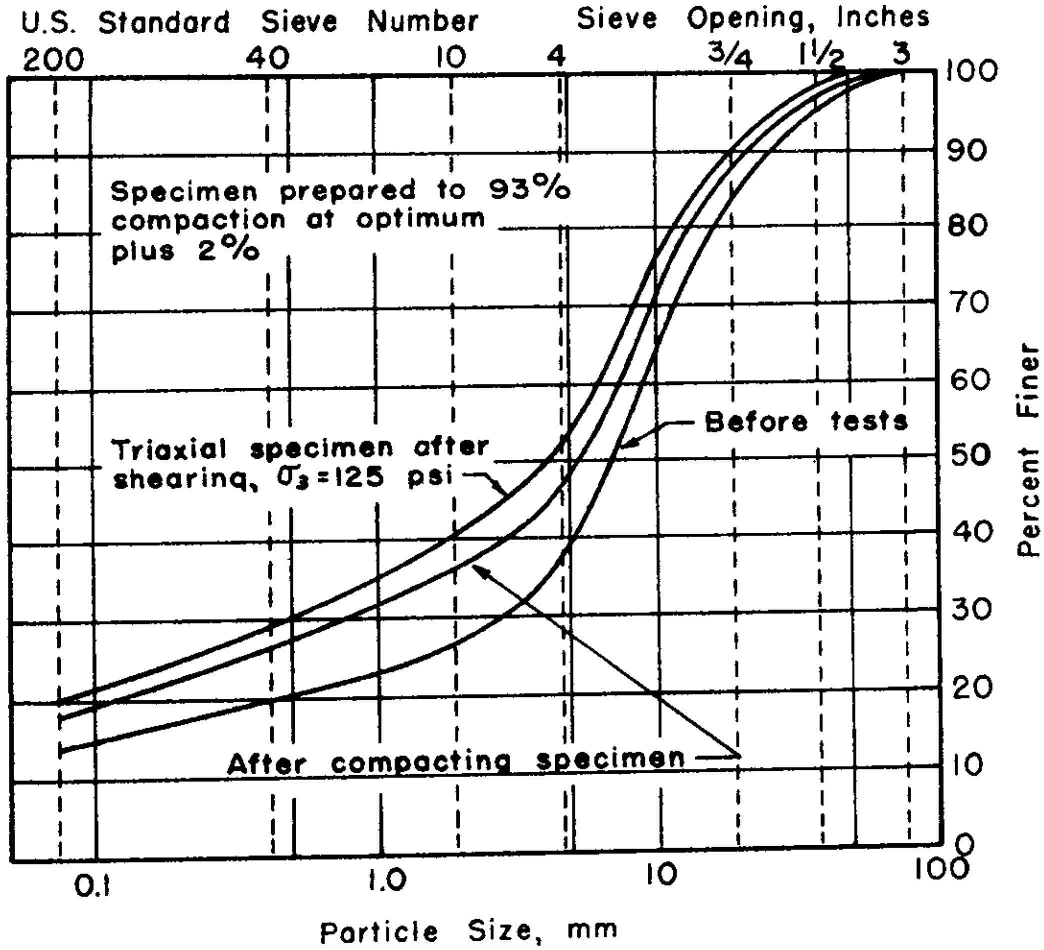

Figure 1. PARTICLE SIZE ANALYSIS OF MINUS 3 INCH MATERIAL "BEFORE" AND "AFTER" TESTS

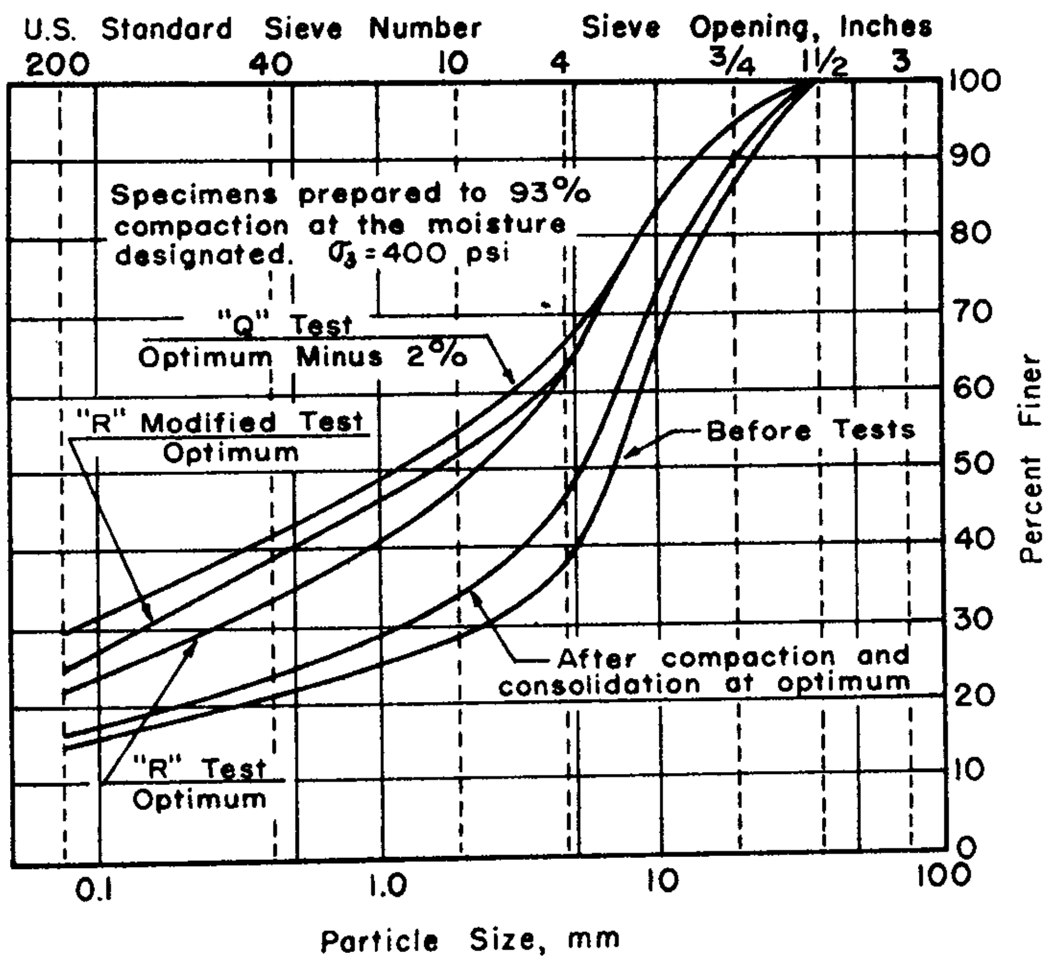

Figure 2. PARTICLE SIZE ANALYSIS OF MINUS $1\frac{1}{2}$ INCH MATERIAL "BEFORE" AND "AFTER" TESTS

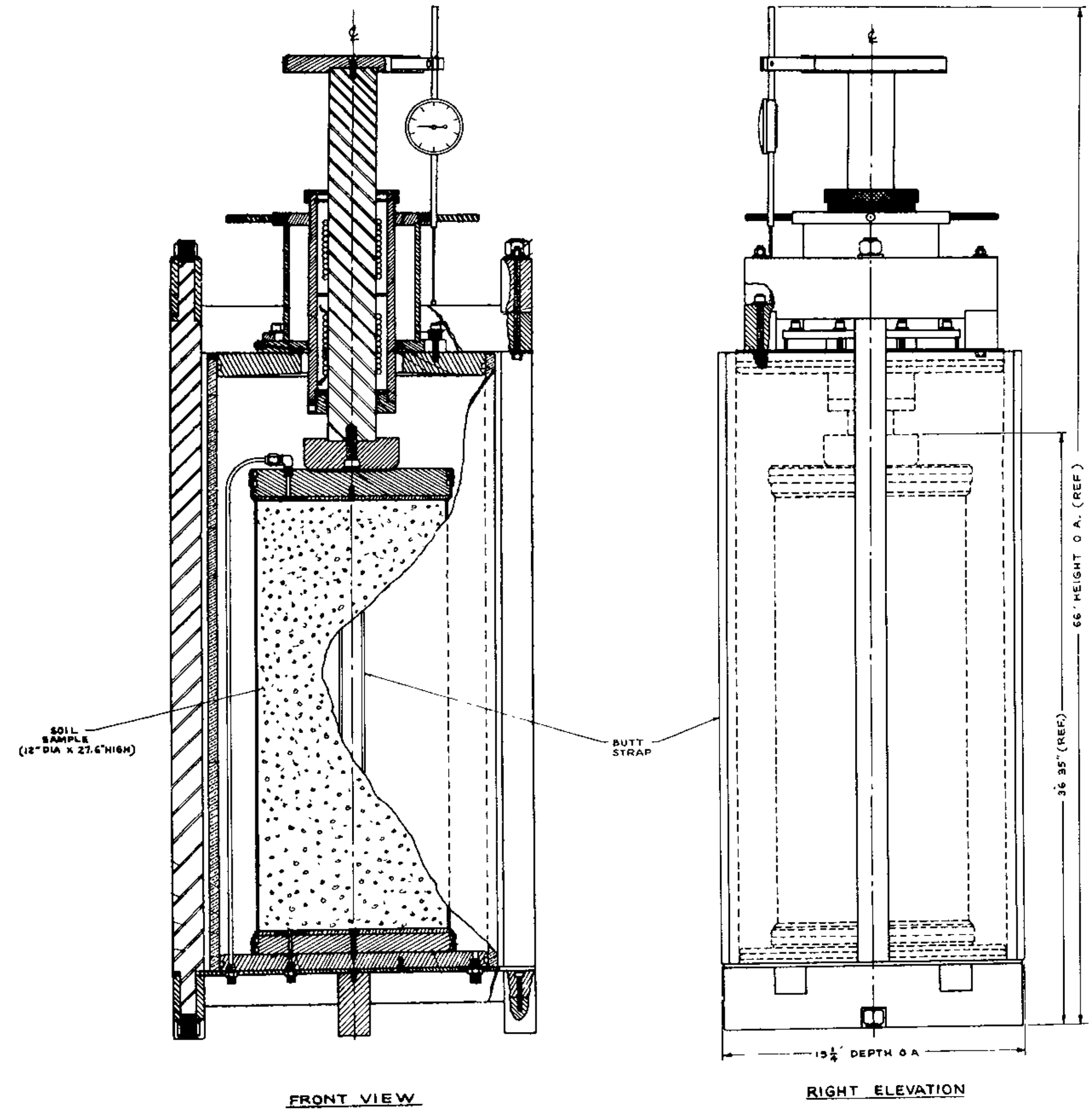

Figure 3. LARGE SCALE TRIAXIAL APPARATUS

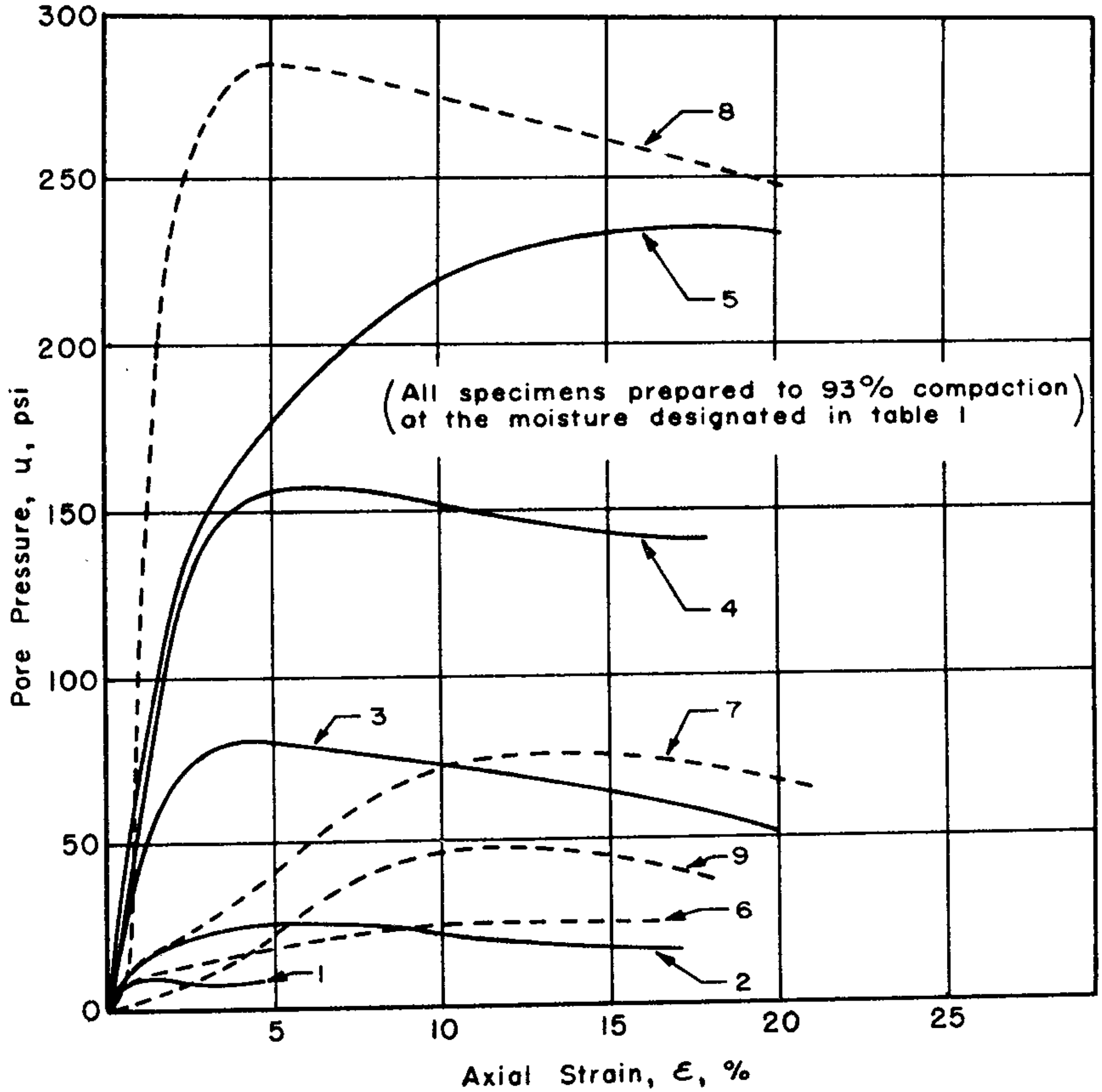

Figure 4. PLOT OF PORE PRESSURE AND AXIAL STRAIN

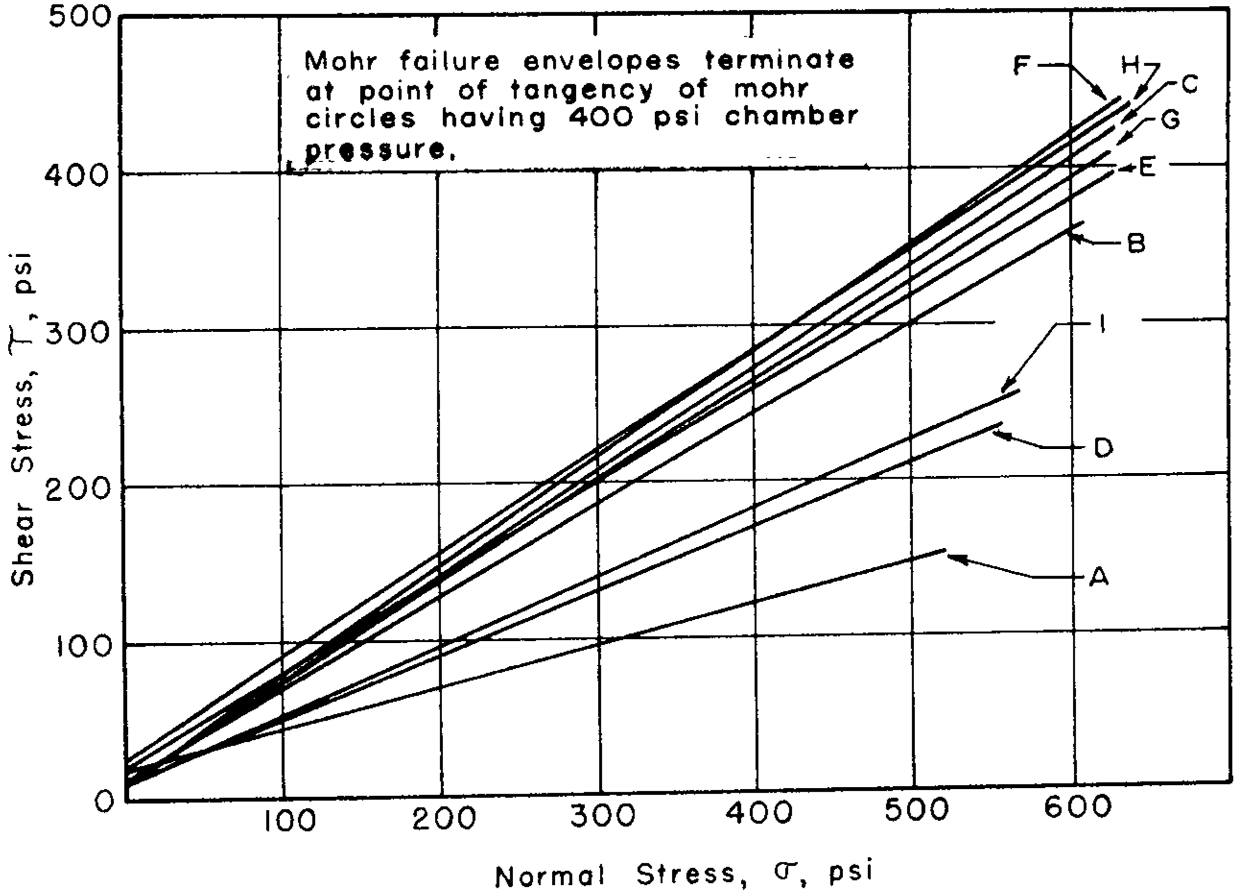

Figure 5. FAILURE ENVELOPES FOR TOTAL STRESSES

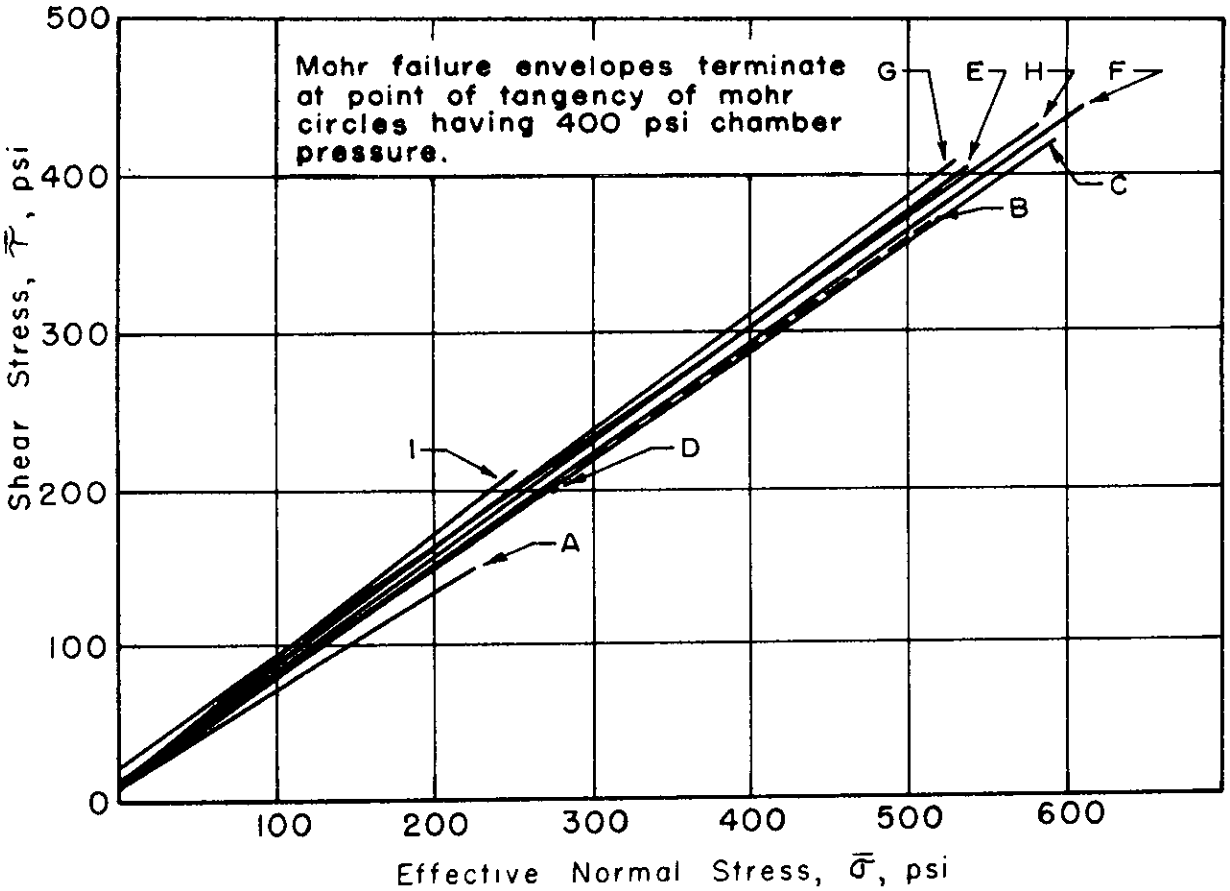

Figure 6. FAILURE ENVELOPES FOR EFFECTIVE STRESSES

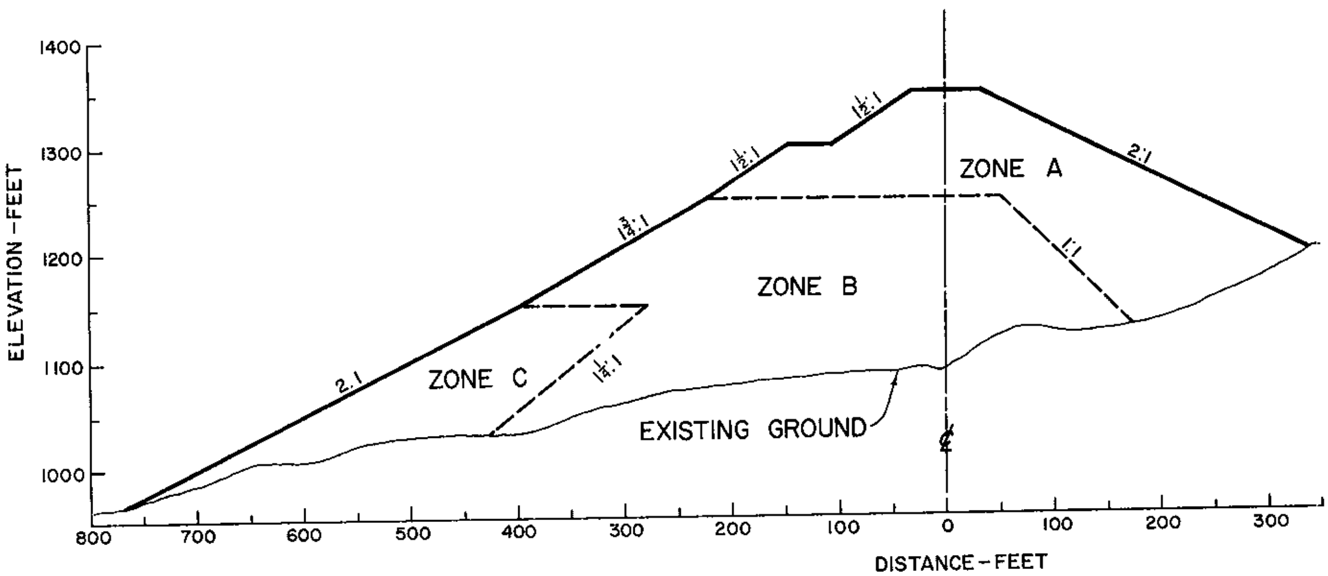

Figure 7. PROPOSED CROSS-SECTION OF EMBANKMENT AT SQUAW CREEK